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MATADERO CREEK REMEDIATION PROJECT
DESIGN CONCEPT

QA/QC ANALYSIS REPORT

September 18, 2002

MARK THOMAS & COMPANY, INC.
Providing Engineering, Surveying and Planning Services

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Introduction

The Matadero Creek Remediation Project proposes improvements to Matadero Creek between Alma Street to the Palo Alto Flood Basin (PAFB). These improvements will provide 100 year flood protection with adequate freeboard to meet the Federal Emergency Management Agency (FEMA) and Santa Clara Valley Water District (SCVWD) Standards.

Prior to this design phase, the SCVWD initiated a planning study to evaluate the conditions of Matadero Creek. Several studies were conducted to evaluate these conditions as they relate to creek hydraulics, geotechnical investigations, hazardous materials, the structures, and the review of the Environmental Impact Report (EIR)/Engineer's Report (ER).

The purpose of this report is to provide the status of the current design project as well as to provide an independent Quality Assurance/Quality Control (QA/QC) review of the various studies. The complete reviews of the separate reports are included as appendices, however, a summary of each review is discussed in the following pages.

Data Gathering and Review, Task 2.1

Data was gathered from the Santa Clara Valley Water District, consultants previously working with the Matadero Creek Planning Study, the City of Palo Alto and various utility companies. The data included as-builts, electronic files, alignments for the upstream reach of Matadero Creek, utility information and the reports written during the planning phase. All of this data was reviewed and organized for reference during the project duration.

Currently, the alignment for the bypass channel downstream of Highway 101 is still under investigation. This alignment needs to be finalized, as it is crucial to the design phase in this downstream reach. In addition, the draft Environmental Impact Report must be certified for the design phase to move forward. Resolution of these issues would not only facilitate the design phase, but would allow the calculation of permanent and temporary easements for right of way acquisition to proceed.

Review Base Mapping and Survey, Task 2.2

Upon review of the existing base mapping, it was noted that there was a conflict between the horizontal control in the electronic files obtained for the downstream and upstream reaches. After contacting the information providers, it was determined that the upstream reach contained survey information in California State Plane Coordinate System NAD 27, whereas the downstream reach contained survey information in California State Plane Coordinate System NAD 83.

Once this discrepancy was identified, the Santa Clara Valley Water District was informed and given a recommendation to translate the upstream survey information from NAD 27 to NAD 83 because the latter represents a more current version of the California State Plane Coordinate

System. The survey information was transmitted to the District's survey staff and was translated accordingly. Upon receipt of the new survey information, the electronic files were translated to the new coordinate system by moving, rotation and scaling the electronic data according to the translation output. With the completion of this translation, the electronic files match up in the same coordinate system.

Compile Base Plans of Existing Utilities, Task 2.3

To compile the maps of the existing utilities, the City of Palo Alto was contacted to provide a list of potential utility companies in the project vicinity. They also provided a list of probable fiber optic companies within the area. To date, the following fiber optic companies have been contacted: MCI WorldCom, Qwest Communications, Level 3 Communications, Sprint Communications, Metromedia Fiber Networks, ICG Telecom Group, Williams Communications Group, Inc., and Nextlink California. Of those contacted, information from Nextlink California and ICG Communications has not been received. In addition, no contact information has been found regarding XO Communications, Infotech Communications and Point to Point. Other utility companies contacted include Pacific Gas and Electric (PG&E), Pacific Bell and AT&T Broadband regarding their facilities within the project area.

Once contact was established with the various utility agencies, a formal request for utility mapping information within the project area was sent. Upon receipt of that information, it was compiled into a set of utility base maps showing the utility facilities within the project area. These base maps were sent as a courtesy copy to the utility owners that may be affected by this project along with a formal request for review of any conflicts. The base maps were also compared to as-builts to verify that all necessary information was included. If there are any discrepancies between different utility information, the utility companies are contacted in order to resolve the issues and the base maps are updated. Utility contacts and resolution of these conflicts are documented in a log system maintained by MTCo, until all conflicts have been resolved and utility data has been updated and confirmed by respective utilities for accuracy.

Hydraulic Analysis Review, Task 2.4

The review of the Hydraulic Analysis was performed by Mark Thomas & Co., Inc. (MTCo), which included the review of the Draft Environmental Impact and Engineer's Report, the Request for a Conditional Letter of Map Revision (CLOMR) and the HEC-2 input/output. In general, MTCo concurs with the conclusions of the Hydraulic Analysis that the floodwalls and headwalls would need to be raised to conform with the FEMA's and SCVWD's freeboard requirements for the 100 year flood event. A summary of the review of the Hydraulic Analysis is presented in the following paragraphs. The entire Review of the Hydraulic Analysis is included in Appendix 1.

Several items in the original Hydraulic Analysis could not be reviewed completely due to lack of information. Some of these items included the following: the base area for San Francisquito Creek; the frequency analysis calculations for San Francisquito Creek and Matadero Creek; the unit hydrograph; background for the SA, SE, and SQ cards for the HEC-1 analysis; the diversion in the HEC-1 analysis; and the Barron Creek cross sections for the HEC-2 analysis.

There were also a number of minor inconsistencies in the original Hydraulic Analysis Report. These included incorrect values in the summary tables or different values listed in the summarizing tables than in the HEC-1 input. The most significant of these differences concerned the HEC-1 analysis for the inflow to the Palo Alto Flood Basin. The HEC-1 results and the results summarized in Table 2 presented some inconsistencies. These inconsistencies stemmed from the UC cards, which determine the time of concentration (t_c) and the R coefficient. Since these UC card values were high in almost all of the Matadero Creek subareas, they therefore effected the timing of the peak of the hydrographs. These values should be reviewed and the corresponding table and model revised, unless there is another explanation for the high values. The RM card values summarized in Table 5 do not match those values used in the model therefore, they should be checked.

The HEC-2 Existing Model of Matadero Creek provided in the original Hydraulic Analysis was only reviewed briefly since the proposed improvements are based on the Post Model. However, when the Existing Model was reviewed, some inconsistencies with the stations were found. For instance, a station identified in the TN card (9316) did not match the station used in the corresponding NS card (9129). Also, a section is listed in the first TN record (9286) that does not exist in the model.

After reviewing the HEC-2 Post Model of Matadero Creek provided in the original Hydraulic Analysis, a few modifications to the modeling is recommended. The review of the cross sections revealed that the width of the channel used in the HEC-2 analysis did not always match the width shown in the as-builts of Matadero Creek, especially at the bridges. Specifically, the multiplication factors used at Greer Road Bridge (.983), Louis Road Bridge (.989), Ross Road Bridge (.968), and Middlefield Road Bridge (.991) adjust the widths of the sections at those locations such that they do not match the widths shown in the as-builts. In addition, the widths of the channel in the sections at Cowper Street Bridge and Waverly Street Bridge also do not match the widths shown in the as-builts. It is recommended to revise the cross sections at the bridges included in the HEC-2 Post Model to more accurately represent the as-built width of the channel.

The review of the model itself identified that the four bridges at the downstream end of the model (the footbridge, the frontage road, Highway 101 and Bayshore Road) were all modeled as separate bridges. This could exaggerate the expansion and contraction losses since they are calculated for each bridge regardless of the fact that they are so close together. Since these bridges are very close together and are similar in cross section, so they could be modeled as one bridge to more accurately represent the losses. However, since the water surface elevation does not touch the bridge soffits and the creek is completely lined, it is recommended that this area be modeled as an open channel to eliminate any uncertainties with respect to these losses arising from bridge modeling accuracy.

The review of the Hydraulic Analysis also recommended that at the Alma Street Bridge where the SPRR culvert oufalls into Matadero Creek, the channel could be modified to reduce losses by designing a more gradual transition.

The HEC-2 Post Model in the original Hydraulic Analysis displayed "cautions" for certain cross-sections within the model, which may represent supercritical flow. These "cautions" indicated that the flow just downstream of Ross Road and Middlefield Road are near critical depth. If a check for critical depth was performed, then more accurate velocities could be determined and potential scour of channels could be addressed. Using the HEC-2 Post Model to check for critical depth may not be worthwhile. However, if the HEC-2 model was converted to HEC-RAS, the check for supercritical flow would be simple. Converting the HEC-2 models to HEC-RAS would take additional time and effort since both the Post Model as well as the Existing Model would need to be converted. In addition, the HEC-RAS models are not always accepted in all jurisdictions. For instance, FEMA has just recently begun to allow HEC-RAS models for Flood Insurance Studies. Converting the HEC-2 models to HEC-RAS is recommended to check for critical depth and therefore, for scour potential.

Geotechnical Design Review, Task 2.5

Parikh Consultants, Inc. reviewed the Geotechnical Investigation dated September 27, 1999 and the revised Geotechnical Investigation dated December 21, 2001, both prepared by Lowney Associates. The Geotechnical Investigation presents the geotechnical conditions and recommendations within the project area. The original investigation acknowledges that the proposed improvements to Matadero Creek are feasible geotechnically, provided their recommendations are heeded during design and construction. Following the review, Parikh Consultants, Inc found that the some of the information presented in the Geotechnical Investigation needed to be clarified and/or supplemented to provide a more complete geotechnical overview of the project. A summary of the Geotechnical Investigation and the review comments is presented below. The entire Review of the Geotechnical Investigation by Parikh Consultants, Inc. is included in Appendix 2.

The original Geotechnical Investigation identified the soil types within the site as 3.5 to 8 feet of fill material overlaying the native alluvium. The soil types were determined from 11 exploratory borings ranging from 25 to 31.5 feet deep. The fill material consists mainly of silty clay with variable percentages of sand and gravel, indicating a high expansion potential. However, the native material consists of stiff to very still silty clay, medium dense to dense sand, medium dense to dense clayey sand, and medium dense to very dense gravely sand. Based on these findings, it was identified that the native material within the site has low to moderate expansion potential. Excavated material with an organic content of 3% or less can be reused as fill. Since the above conclusions were based on relatively small diameter, widely spaced borings, the Geotechnical Investigation recommends that the earthwork and foundation preparation be performed under the observation of the geotechnical engineer.

The site does not occur within a designated Alquist-Priolo Earthquake Fault Zone (Special Studies Zone) and surface expressions of active faults are not believed to exist within the site.

Therefore, fault rupture is not anticipated through the site. Since there is the potential of earthquakes within the general region, some ground shaking is anticipated. However, because the top portion of the soil consists of clays, there is a low potential for liquefaction. In addition, the concrete channel walls retain the soil, so the probability of lateral spread is also low.

Based on the findings, the Geotechnical Investigation provides the following recommendations regarding the concerns of the influence of groundwater, the support of temporary excavations and the presence of moderate to high expansive potential. The contractor should anticipate using a means of ground water control, such as dewatering, during the construction of the proposed improvements. The dewatering systems should be properly designed to prevent pumping the soil fines with the discharge water. The contractor should sample and test the groundwater for soil fines content from the dewatering wells, as needed. The contractor should develop an excavation support plan to provide safe working conditions and to reduce the potential of damaging existing utility facilities. To alleviate the problem of highly expansive soils, the Geotechnical Investigation recommends that slabs-on-grade be sufficiently reinforced, be supported by non-expansive fill, and that the foundations be placed below the depth of seasonal moisture fluctuation.

Ground water is highly anticipated during excavation for the proposed improvements on Matadero Creek. For this reason, the Geotechnical Investigation stresses the importance of having a plan for controlling groundwater, such as a dewatering plan in areas of excavation for the new bypass channel and for the U-Frame Section Channel construction. Upon review of the Geotechnical Investigation, Parikh Consultants, Inc. recommends that the ground water elevations be verified since the groundwater depth did not correspond with the NGVD 29 elevations.

The Geotechnical Investigation recommends that temporary slopes and shoring be designed by a registered civil engineer and performed by the contractor under the strictest governing safety standards to sustain the lateral soil pressure plus additional surcharge. These temporary systems need to be monitored by the contractor for movement and settlement of walls during construction. If movement or settlement is observed, appropriate actions countering the movement should be taken. Various shoring schemes, such as sheet piling, soldier piles or lagging-braced shoring may be used to temporarily shore vertical excavations.

During excavation, any exposed surface soils that are to receive fill, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned and compacted to 90 percent relative compaction. Due to the moisture sensitivity of clayey soils, compaction may be difficult and the materials may need to be dried or stabilized. Fill should be in lifts of no more than 8 inches of uncompacted thickness, be at a moisture content near optimum and be compacted to at least 90 percent relative compaction. Where pavement is placed, the aggregate base and the upper 6 inches of subgrade should be compacted to at least 95 percent. In addition, heavy equipment should be restricted where the depth of water table is less than 2 feet from the grading plane. Adequate drainage must also be provided.

The Geotechnical Investigation recommends that the proposed floodwalls be supported on drilled pier foundations for a minimum diameter of 16 inches and a minimum depth of 8 feet below the existing ground surface. The design allowable skin friction was recommended as 500 pounds per square foot with a one-third increase allowed for either transient wind or seismic loading. Following the review of the Geotechnical Investigation, it is recommended that the derivation of the allowable skin friction be described in the report.

During the initial review of the Geotechnical Investigation, it was recommended that a discussion regarding "spread footing with a seepage analysis" be added to the report. In response, an extra section titled "Spread Footing Foundations for Floodwalls" was added to the Geotechnical Investigation dated December 21, 2001. However, Parikh Consultants, Inc. suggests that it be stated whether the calculation of lateral earth resistance can include both the friction between the footings and the supporting grade combined with the passive resistance.

Under the Louis Road Bridge, a U-Frame Channel Section will be constructed up to 3 feet below the existing channel. A mat or channel bottom slab extending the full width overlaying 24 inches of permeable material is being considered for supporting the channel. This mat or slab will need to be designed for average allowable bearing pressures and lateral earth pressures. During review of the Geotechnical Investigation, Parikh Consultants, Inc. recommended that the uplift resistance on the U-Frame Channel Section from the ground water should be considered in the analyses. Since the groundwater was measured at 7.5 to 13.5 feet below the existing ground and the U-Frame Channel will be positioned up to 3 feet below the existing ground, the uplift forces are expected to be significant.

Concerning the Lateral Earth Pressures, the original Geotechnical Investigation recommended that "the walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H psf, where H is the distance in feet between the top of the footing and the top of the wall." The review of the Geotechnical Investigation recommended that the lateral earth pressure loads and calculations be clarified further. Specifically, the report should explain whether the load distribution is a "trapezoidal load distribution for the at-rest soil condition." Also, the derivation of the uniform pressure requirement of "8H" should be included in the report.

According to the Geotechnical Investigation, it may be preferable to adopt 24-inch diameter Cast In Drilled Hole (CIDH) piles as the new foundation for the proposed Louis Bridge Structure because these piles will minimize the disturbance of utility facilities and aid in constructability. The Geotechnical Investigation also recommended that the piles extend 30 feet below the bottom of the pile cap for 45 ton piles and 40 feet below for 70 ton piles where the pile cap is no deeper than 5 feet from the existing or proposed ground. After reviewing this section of the original report, it is recommended that additional discussion be provided to fully investigate the foundation for Louis Road Bridge. These items would include: calculations to justify the vertical pile capacity; soil profile and strength parameters for the estimation of the pile response under lateral load; cut-off and tip elevations for the piles; deeper borings for the 70 ton pile (with the approximate depth of 45 feet); and more detailed information regarding the computer analyses of lateral loads. The review also questioned the significance of the phrase, "where the pile cap is no deeper than 5 feet from the existing or proposed ground" with respect to the pile design. In addition, corrosion on the foundation piles due to native soil should also be investigated and included in the report.

Parikh Consultants, Inc. also reviewed the Geotechnical Investigation prepared by Lowney Associates, dated September 27, 2001 for the Matadero Creek Overflow By-pass Channel at the MSC.

Four exploratory borings were taken by Lowney Associates in July 2001. This information along with four supplemental logs from previous exploratory borings showed that a fill layer exists over a soft to medium stiff clay over stiff sandy clay and clayey sands. This fill thickness at the boring sites ranged from 9 to 11 feet thick. The medium still clay ranged in thickness from 3 to 4 feet. The fill across the site can be generally classified as still to very stiff, moderately to highly plastic clays, with a estimated underderain shear strength ranging from 1500 to 4000 pounds per square foot. The medium stiff clays appeared to be Bay mud but the soils did not exhibit the very soft and highly compressible properties of a typical Bay mud. This layer consists of a medium stiff, highly plastic, blue-grey silty clay with an estimated underdrain shear of 1000 pounds per square foot. The soils below the medium stiff clays can be classified as silty clays, sandy clays and medium dense clayey sands with an underdrain shear strength ranging from 1500 to 3000 pounds per square foot.

Similar to the upstream reach, the contractor will encounter groundwater, at the By-pass Channel, at depths ranging from 14 to 15 feet which corresponds to -5.25 and -6.25 feet 1988 NAVD, respectively. According to the Geotechnical Investigation, the groundwater elevations can fluctuate as much as two feet due to tidal actions, rainfall, underground drainage patterns and other factors that could not be identified at the time the groundwater measurements were made.

Since groundwater will be encountered during the construction of the By-pass Channel, the contractor needs to be aware of the potential for groundwater fluctuation and appropriate dewatering systems will be required during construction. All such dewatering systems should be properly designed to prevent pumping the soil fines with the discharge water. The contractor should sample and test the groundwater for soil fines content from the dewatering wells, as needed. The contractor should develop an excavation support plan to provide safe working conditions and to reduce the potential of damaging existing utility facilities.

The site is not located in a designated Alquist-Priolo Earthquake Fault Zone. There are no surface expression of active faults, therefore fault rupture is not anticipated through the site. Similar to the upstream reach, there is the potential of earthquakes within the general region, so some ground shaking is anticipated. However, the top portion of the soil consists of clays, and therefore, there is a low potential for liquefaction. In addition, the concrete channel walls retain the soil, so the probability of lateral spread is also low.

Heavy equipment will be utilized during the construction of the By-pass channel. As such the contractor shall be made aware of the sensitivity of clayey soils and compactions difficulties. According to the Geotechical Investigation, heavy equipment should not be allowed to operate on the excavations made to an Elevation of 4.75 (88 NAVD) since this zone contains the potentially unstable material which may lead to deflection and "pumping" of the soil. It is recommended that this zone should be overexcavated to a depth of 3 feet and replaced with bridging rock and a geotextile fabric. Any proposed over-excavation at the bottom of the By-pass channel should also be considered in the slope stability evaluation.

The existing soil, with the exception of Bay mud, below the stripped layer may be used for backfill material. In general, the fill material shall not contain rocks or lumps larger than 6 inches in greatest dimension. Structural fill shall be inorganic and should have a Plasticity Index of 15 or less and a Liquid of 40 or less. All fill used for slab-on-grade shall be compacted to a minimum of 90 percent relative compaction (ASTM D1557). Fill shall be placed in lifts no greater than 8 inches. For pavement areas, fill used in the upper 6 inches and full depth aggregate base shall be compacted to 95% relative compaction (ASTM 1557). Where the backfill compaction will require heavy equipment, it may be more prudent to utilize backfill material that is "self compacting" or requires minimal effort such crushed rock.

The project description discusses the use of an interlocking block type gravity wall but the "backfill" section discusses the placement of wall reinforcement and its length. The wall reinforcement generally relates to a MSE (Mechanically Stabilized Embankment) type of wall which is internally stable. This should be clarified. During the Value Engineering review it was suggested to use Secura Slope interlocking block wall. As a result, the wall reinforcement will not be used. Instead, geogrid will be used along the length of the wall to "tie back" the interlocking to the bank. As of September 17, 2002, the SCVWD requested to lay back the block wall to minimize the amount of overexcavation of the channel bank and possibly eliminate the use of the geogrid or helical anchors. Lowney and Associates are reviewing the design parameters.

The Geotechical Investigation proposes to use temporary interlocking sheet piles. These temporary sheet piles shall be placed by the contractor in accordance to the strictest governing standards. However, if the contractor wishes to remove the sheet piles, he must demonstrate an effective yet cost efficient method to remove the sheet piles without disturbing the channel or the MSC.

The soil structure interaction of the interlocking block wall should be more elaborate in the report. Will the contractor or the engineer be responsible for the design? Are the blocks facing? Will the blocks provide lateral capacity and if so, what happens if they shift? Additionally, the interlocking blocks uses a permeable material for drainage. It is Parikh Consultants, Inc. opinion that even with this material, it is prudent to use a filter fabric to protect the fines from plugging the backfill or washing away of the fines due to the "suction effect" from the water.

As a result of changing the type of interlocking block that will be used in the project, the design parameters of the wall must be modified from the original Engineer's Report. The new parameters were supplied by the manufacturer and Lowney Associates was subcontracted to prepare an independent report to confirm the parameters.

Hazardous Material Review, Task 2.6

Geocon Consultants, Inc. reviewed two hazardous material reports as part of the preparation of the Planning Study for the Matadero Creek Remediation Project. These reports are titled "Phase 1 Environmental Site Assessment and Soil and Ground Water Quality Screening" and "Additional Soil and Ground Water Quality Evaluation". The reports were prepared Lowney

Associates. The Hazardous Materials reports identify the hazardous material, contaminated locations and contaminant levels. In general, Geocon Consultants, Inc. agreed with the findings of the Hazardous Materials reports, but recommend that the contamination limits not be identified to the contractor. A summary of the review of these two Hazardous Materials reports is presented in the following section. The complete review is provided in Appendix 3.

The Phase 1 report identified a location on Middlefield Road where a source of petroleum hydrocarbon was encountered in the soil, during a creek improvement project. It was presumed that the source was a leaking fuel storage tank from an existing service station.

As a result, it is anticipated that a level of residual petroleum hydrocarbon may exist in and around the area that may affect the handling of the soil during the construction of the project.

Four other locations were identified as possible contamination sites. The first two sites, the 842 Wintergreen Avenue and the Megher Property, were listed in a database report but it was not clear if the contamination status of these properties had changed. No further investigation or discussion of these properties was cited in the report.

The final two locations identified as possible contamination sites are two dairies – the Piers Dairy and Old Pier Dairy. The Phase 1 report does not identify whether the source of contamination is due to runoff of the property, elevated nitrates, solid waste disposal or petroleum hydrocarbon contamination on the site. No further discussion of these properties was cited in the hazardous materials report.

The "Additional Soil and Ground Water Quality Evaluation" report inspected the area downstream of Highway 101. This report identifies the Palo Alto Municipal Service Center as a petroleum hydrocarbon contamination site, based on ground water sampling results in and around the MSC.

The hazardous material reports identify several possible contamination sites that may affect the construction of the Matadero Creek Improvements. To minimize the number of Contract Change Orders, it is recommended by Geocon Consultants, the independent reviewer, that the contractor(s) be advised that they may encounter residual levels of petroleum hydrocarbons in the soil during construction. As such, warning notes informing the contractor(s) of the contamination should be added to the plans and in the specifications. It is also recommended that the contamination limits not be shown on the plans, in the event that the contractor(s) encounter impacted soil/groundwater beyond the stated limits.

The previous section summarized the information presented in the Hazardous Materials reports and the corresponding review comments. Refer to Appendix 3 for further clarification of the specific review comments of the Hazardous Materials Reports.

Structural Review, Task 2.7

Mark Thomas & Co. (MTCo) conducted a review of the Structural Investigation Report as prepared by Biggs Cordosa Associates in June 2001. The Structural Investigation presents the current status of the structures within the project area as well as identifies the proposed project improvements from a structural standpoint. In general, the conclusions regarding the structures, as stated by Biggs Cordosa Associates in the Structural Investigation, appear appropriate, however, we noted some discrepancies between existing field conditions and assumptions made in the Investigation. The following section summarizes the information from the Structural Investigation and the corresponding review comments.

The Structural Investigation describes two options for the bypass channel downstream of Highway 101: concrete lined bypass channel and concrete block lined bypass channel. Construction would consist of driving temporary sheet piling, installing drain rock, constructing a concrete invert slab, constructing concrete or precast concrete block lined bypass walls, removing temporary shoring and planting the area adjacent to the MSC. The preliminary design assumptions for this construction state that ground water elevation is 0.0 to 2.0 and that bypass layout and sizing would be based on a rigid and undrained wall system or manufacturer design charts. Assumptions also included the following: long-term buoyancy effects are negligible, short-term buoyancy effects are not as significant because the structure is not watertight, the top of wall elevation is 7.0 and the preliminary review focused on the typical bypass section. Based on our review of the Structural Investigation, these assumptions and concepts are appropriate to proceed with the final design and no changes are recommended.

Using appropriate design assumptions, the bridges upstream of Highway 101 were analyzed as functioning under pressurized flow by assuming that the design water surface includes the available freeboard. From this analysis, the Louis Road Bridge is considered significantly overstressed for the one percent flood and therefore, must be raised. The trapezoidal channel in the vicinity of Louis Road Bridge must also be replaced by a U-Frame Channel Section to allow additional flow. Even though the Demand/Capacity ratios presented by Biggs Cordosa in the Structural Investigation could not be verified during the review process, the conclusions were deemed appropriate.

Approximately 14,500 linear feet of floodwalls of all different types extend along Matadero Creek. During the verification of the structural conditions presented in the Structural Investigation, it was determined by MTCo that there were few discrepancies between the report and the field conditions. For example, the Structural Investigation describes the existing and modified floodwalls as being on spread footings, whereas it was observed in the field that these wall types were built with steel posts, 8 feet on center. It was also stated in the Structural Investigation that Type 1 floodwalls were not constructed, but were replaced by Type III floodwalls. However, according the as-built plans and the field conditions, it was determined by MTCo that Type I floodwalls do exist in the project area. In addition, the Structural Investigation states that Type 4 floodwalls were not used. However, further review of the field conditions verified that Type 4 floodwalls are indeed present along Reach 3 adjacent to Louis Road. These discrepancies will be corrected and will be shown correctly per our field investigation and verification. Preliminary analysis performed during the review still places these walls under the "Buttress or replace floodwall" improvement category as described below.

According to the Structural Investigation, improvements to the floodwalls fall into three categories: (1) sections of floodwalls that must be raised without additional support, (2) sections that must be replaced or strengthened and (3) sections that must be replaced in conjunction with the proposed channel lining modifications at Louis Road. Raising a floodwall would consist of cast-in-place concrete wall extensions doweled into the top of the existing Type 7 and Type 8 floodwalls. A floodwall would be replaced by cast-in-place concrete wall on spread footings or span horizontally between cast-in-place concrete pilaster extensions of cast-in-drilled-hole piling. If only strengthening was required, the existing floodwall would be buttressed with cast-in-place pilaster extensions of cast-in-drilled-hole piling. It is recommended that this metholodogy for extension satisfies the FEMA seepage requirements for floodwall along Reach 2 where the wall footings are not protected by the channel lining. If the final design that the seepage may be a problem, the proposed wall replacement details would be revised to incorporate seepage control measures.

Since several of the bridge structures are anticipated to experience pressurized flow during the one percent flood, the headwalls at Greer Road, Ross Road, Louis Road and Waverly Street will need to be raised to meet FEMA's freeboard requirements. The headwalls consist of concrete barriers with tubular handrailing. To raise the headwalls, the existing railing would be removed and a cast-in-place concrete wall extension would be doweled into the existing headwall. The Greer Road Bridge headwall will need to be further strengthened or replaced because it will not have the capacity to support the increased hydraulic loading. It is suggested that a "more integrated form of the detail shown in the Structural Investigation Report" be used to extend the headwalls. In addition, it was determined that the headwall extension at Greer Road Bridge will need to be approximately two feet greater than that described in the report. It is therefore recommended that an alternative modification detail be developed or the entire headwall be replaced.

The maintenance access gates will also need to be modified to provide the appropriate flood protection. Two solutions were presented in the Structural Investigation: (1) raising the existing ramp elevations or (2) installing a system of semi-permanent gates. Raising the elevation of the ramps would either increase the ramp slope or lengthen the ramp to accommodate the elevation difference. Therefore, MTCo agreed with the Structural Investigation that the system of semi-permanent gates would be viable and eliminate the need for increased ramp slopes.

The Structural Investigation also mentions the importance of protecting the two 60 kV underground electric lines because they are high risk utilities. Protective measures include that any structure, temporary or permanent, should not be within two feet of the existing 60 kV trench and that construction operations should minimize vibrations or other impacts to the utility lines.

Appendix 4 contains more detail information regarding our review of structural elements.

Environmental Impact Report/Engineer's Report (EIR/ER) Review, Task 2.8

MTCo reviewed the Environmental Impact Report (EIR)/Engineer's Report (ER) as presented by the Santa Clara Valley Water District. The EIR/ER describes the project, identifies environmental impacts, the findings and mitigation. MTCo agrees with the findings of the EIR/ER, but recommends clarification of certain sections that contained discrepancies, which are as follows:

The EIR report identifies the project, the project site, the project objectives and how the achieve those objectives. The specific objectives include: increasing the flood capacity to carry the one percent flood, providing equal flood protection, protecting the habitat within the project area, providing efficient and cost-effective flood control, removing existing areas from the flood insurance map and reducing the risk of tidal flooding. It is recommended that the EIR clarify the extents of flood relief granted from the project improvements.

The description of the recommended project includes a bypass channel downstream of Highway 101, which would be constructed along the MSC. The bypass channel would consist of a gravity retaining wall system and a hard bottom that runs along the edge of the MSC. The channel would be approximately 60 feet in width and have a 10-foot construction buffer zone on either side of the channel. It is recommended that the descriptions and the exhibits of the proposed bypass channel be consistent throughout the EIR/ER and display all of the proposed aspects of the bypass channel downstream of Highway 101.

One proposed enhancement along the downstream reach of this project was to include a trail between the floodwall and the MSC fence line. During the review process of the EIR/ER, it was noticed that the floodwall mentioned in this enhancement section was not appropriately described. Since this floodwall is apparently necessary for flood protection of the MSC, it should be discussed in more detail in the other sections describing the downstream bypass channel and not solely in the enhancement section. It is also suggested that the location of the trail be further clarified, as it is unclear if this trail would be within the 10-foot construction buffer zone.

The upstream reach included improvements such as raising floodwalls and headwalls between Highway 101 and Alma Street, raising the Louis Road Bridge and converting the channel from a trapezoidal channel to a U-Frame Channel Section in the vicinity of Louis Road. Since the proposed floodwalls and headwalls will be approximately 7 feet above the sidewalk area, the visual impact should be minimized through the usage of artwork and decorative architectural features. The detail of aesthetic treatments would be finalized during the final design phase with community input and review process.

The District is in the process of applying to FEMA for relaxed freeboard requirements. If approved, the floodwalls between Greer Road and Louis Road could be reduced to 5.3 feet. There is a statement in a different section of the EIR/ER stated that the floodwalls could be "reduced by up to 5.3 feet in the most optimistic case". It is recommended that the discrepancy between these two statements be resolved.

We have also noted that the report contains conflicting statements identifying the length of channel to be modified. In one case, the EIR/ER states that 65 feet of existing trapezoidal channel directly under Louis Road will be converted to U-Frame Channel Section with a transition on both sides of the bridge. Since it is recommended in the EIR/ER that the downstream transition occur within 20 feet and the upstream transition occur within 40 feet, the total length of the proposed modification would be 125 feet. However, another section of the EIR/ER describes the modification as occurring along approximately 120 feet. Even though these numbers are not significantly different, it is recommended that all information be consistent throughout the EIR/ER.

Another element of the proposed project is to construct a new warped transition on the south channel bank at the downstream face of the Alma Street/SPRR culvert and improve the existing 84-inch storm drain outfall. It is recommended that this improvement be discussed in the Environmental Impact Report rather than solely in the Engineer's Report.

After the proposed project was described, the environmental impacts of the project were identified. The EIR identified several plans and policies and stated the consistency of the project with these policies. The land use impacts were then identified as less than significant. It was also determined that the project would provide beneficial impacts on both drainage and flooding and that the geological impacts could be reduced to less than significant if mitigation measures were implemented. The vegetation and wildlife within the project area was described thoroughly and special species were identified in the EIR/ER. Impacts to these areas would need to be mitigated at the appropriate ratio, either on or off site, to be accepted through the permitting agencies. Water quality, air quality, noise, utility facilities, maintenance practices and hazardous material are all concerns within the project limits and would also need to be mitigated to prevent significant impacts.

This section provided information regarding the review of the EIR/ER by the consultant. However, since this report must be publicized to residents and public groups, who may be interested in the project, MTCo was not the sole reviewer of this report. Therefore the issues discussed above correspond to this task and do not include specific comments from other independent reviewers. Other comments and concerns addressed issues regarding mitigation, the downstream bypass channel, backwater effects and more. All of these comments and concerns will need to be addressed prior to certification of the EIR/ER.

A summary of the information presented in the EIR/ER and the review comments were discussed in the previous paragraphs. Refer to Appendix 5 for details regarding the specific review comments of the Environmental Impact Report.

Conclusion

The Matadero Creek Remediation Project proposes improvements to satisfy freeboard requirements for the 100 year flood event. The purpose of this report was to perform quality control and assurance review of the previous studies for consistency, sufficiency of design assumptions and criteria, and confirmation of the adequacy of design concept to proceed to final design.

Our review found minor discrepancies and inconsistencies, which have been noted in previous sections with suggestions for correcting these discrepancies and inconsistencies. Our review also included analysis of the sufficiency of design assumptions, criteria and adequacy of the design concepts. Based on our review, we recommend that the project proceed to final design, subject to satisfying the conditions imposed by this report.

APPENDIX 1

HYDRAULIC ANALYSIS REVIEW

MARK THOMAS & COMPANY, INC.
Providing Engineering, Surveying and Planning Services

Mark Thomas & Co. Inc.
Consulting Civil Engineers & Municipal Planners
1243 Alpine Road, Suite 222, Walnut Creek, Ca. 94596
(925) 938-0383
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Project No. 22-51004

Memorandum

To: Steve Machida

From: Mark Boucher

CC: Richard Tanaka, Mike Lohman, Chuck Anderson (Schaff & Wheeler)

Date: January 7, 2002

Re: Matadero Creek Hydrology QA/QC

I have reviewed the documents you provided which included the following:

- Draft Environmental Impact Report and Draft Engineers Report
- Request for a Conditional Letter of Map Revision (CLOMR) - Matadero / Barron Creeks Redemption Project Volumes II and I.

My review entailed:

- Becoming familiar with the DEIR, DER, and CLOMR
- Performing a detailed review of information included in the CLOMR.
- Review of the HEC-2 input/output in Form 4, Item 2 Attachment - MATPOST.OUT

The items of which I have the greatest concern are bolded and underlined herein.

Specific comments are as follows:

CLOMR Volume I - FORM 3

1. The CLOMR calculations use 37.0 square miles (sqm) for San Francisquito Creek area. I could not find a hydrology map with areas for checking this area or any of the sub water shed areas or impervious area conditions
2. A spot check of the tables of calculations on pages A3.5 and 6 found an error. The table on A3.6 shows three diversion values of 157 cfs in a row. These values should have each been 112 cfs. The result is a higher "Adjusted flow at Gage" for each of them and an increase in the highest Adjusted flow at Gage from 1201 to 1204. This is minor change and would have no great effect on the overall calculations.
3. The capitalization of variables "N" and "n" is switched between the equations on A3.6 and the following graph. Most likely the calculations are correct. However, the values are very

close (45 and 42 respectively and could easily be transposed). Schaaf & Wheeler should verify that the numbers were not transposed.

4. I followed the calculations presented in the CLOMR FORM 3 section using the USGS Bulletin #17 B - "Guidelines for determining Flood Flow Frequency" and found no inconsistency between the CLOMR and the bulletin. I did not have the raw data to perform a complete independent check of the frequency analysis calculations for San Francisquito Creek and Matadero Creek.

5. The CLOMR provided calculations for unit hydrographs using watershed "n" values of 0.100, 0.075, and 0.050. The calculations for the calibration of the Antecedent Moisture Conditions (AMC) in the first HEC-1 run after page A3.17 used a unit hydrograph for a basin with an "n" value of 0.080. I cannot verify that the unit hydrograph in the HEC-1 stack is accurate. Looking at the unit hydrograph values, it appears that they are between those in the runs with "n" values of 0.100 and 0.075 and therefore could be correct. It would be good to verify the unit hydrograph for n=0.080.

6. The results of the first HEC-1 run show a peak flow of 7,711 cfs - greater than the 7,700 cfs stated on page A3.17 earlier in the CLOMR. I could not find the 7,711 cfs value anywhere in the supporting calculations or text. It could be that the difference is due to a simplifying round off practices.

7. The results of the second HEC-1 run (calibration using the Clark unit hydrograph method) shows a peak flow of 7,969 cfs; the same as that stated on page A3.17.

8. The table on page A3.17 shows the $R/(R+t_c)$ as 0.54 in the last row. I calculate 0.53 (rounded from 0.5319). This is a minor difference, and likely insignificance or due to their rounding of the t_c and R values.

9. Review of HEC-1 for Inflow to Palo Alto Flood Basin

   a. I compared the "Watershed Subarea Map", the "Rainfall-Runoff Model Schematic", and the HEC-1 output "Schematic Diagram of Stream Network" and found them to be consistent. The "Rainfall-Runoff Model Schematic" does have a confusing line from node 'C' to node 'D' which was probably drawn to reflect the diversion 'MDIV'. There is a separate 'MDIV' diversion already indicated on that figure.

   b. Several tables provide backup for the model input. I compared these with the model. The Table 2 has HEC-1 Card identifiers at the top making it easy to correlate the model to the input.

      I found inconsistencies between Table 2 and the HEC-1 input. These inconstancies were found in the UC cards which set the time of concentration (t_c) and R coefficient for each subarea and are in units of hours. Both t_c and R effect the timing of the peak of the hydrograph. These coefficients were high in the Adobe Creek watershed (A3a), the Barron Creek watershed (B1 and B2), and in the Matadero Creek watershed (all but M6 and M 12). If the UC card values were high in just a few subareas scattered throughout the model, there would be not need for concern. Since the coefficients are high in almost all of the Matadero Creek subareas, there is cause for concern. **These values should be reviewed and the table or model revised. There may be some other explanation as to why the values are different. If so, the explanation should be presented.**

   c. Some of the RM card values in the model did not match the RM values in Table 5. RM is used for entering the Muskingum routing coefficients. The coefficient in question is the Muskingum K (AMSKK in hours). The routed reaches in question are QtoW (Adobe/Barron watershed), AtoB, BtoC, CtoGage (Matadero watershed) The AMSKK value for QtoW was set equal to the time of concentration t_c. The AMSKK value for the reaches identified in the Matadero watershed were all lower than shown on Table 5. These number should be checked.

d. Other routing input could not be checked. The background for the SA, SE, and SQ cards was not provided in the CLOMR package.

e. The diversions also could not be checked because there was no backup provided for the input.

f. There are a couple of warnings in the output related to the routing. Warnings are not a sign that the model is wrong, but only that certain parameters, either input or calculated by the model, are in a range of instability. Adjustments to remove these warnings are likely not to make a significant difference in the modeling results since they are related only small part of the total model.

g. Overall, the order of the data records is correct and the HEC-1 model is well organized and easy to follow.

CLOMR Volume I - FORM 4

10. Review of HEC-1 for Inflow to Palo Alto Flood Basin

a. The analysis of the tide cycles appears to be thorough. The use of the "storm surge" concept is reasonable and appropriate and results in a more methodically thought out and could be more conservative than using the MHHW usually used by the Corps and FEMA (Par. 1 pg. A4.1-15).

b. I reviewed the flood basin computer simulation spreadsheets and made the following observations:

1. noted that Matadero inflow hydrograph matched the HEC-1 output.
2. I found the volume calculations to be accurate as well as the tide gate flows.
3. On page A4.1-14, the CLOMR states,

"The peak stage that would be observed during such an event is predicted to be 6.5 feet NGVD, which would be limited to 6 feet NGVD by levee overtopping".

I took this to mean that the levee elevation is 6 feet NGVD and the PAFB could not fill higher than that without overtopping. However, in the simulation, the elevation of the tide and PAFB stage are at times higher than 6 feet. Chuck Anderson explained that there is another higher levee which keeps the tidal waters from spilling into the PAFB and reducing the storage for runoff.

There are three tide cycle simulations:

(1) Design Tide Cycle,
(2) Max Stage Tide Cycle, and
(3) FEMA Tide Cycle.

In (1) and (2) the same design tide cycle is used. I understand from the report that (2) is the situation where the timing of the tide cycle was shifted to where its timing with the storm hydrograph restricted the tide gate flows and creates the most back up of water in the PAFB.

In (2) the tide elevation rises above 6 feet twice for about 3.5 hours before the PAFB stage first rises above 6 feet. In (2) the second high tide partially overlaps the time period when the PAFB stage is over 6 feet. This happens at around 46 hours into the event.

If the additional levees were to fail, the PAFB would loose storage capacity and the beginning water surface elevation used it the HEC-2 modeling would be invalid.

11. Review of HEC-2 Barron Creek

    a. I reviewed the HEC-2 input and output for Barron Creek. In checking the flows against the HEC-1 model, I found that the flow rates were rounded up and down in the HEC-2 Model. Most of the rounding is insignificant.

    b. There were no plans for Barron Creek against which to compare the model cross sections. There is a water surface elevation (WSE) profile provided near the back of the FORM 5 section which indicates that the WSE clears all of the bridges except the Louise Road Bridge.

    c. I do not have plans that would allow me to review the cross sections.

12. Review of HEC-2 for Matadero Creek

    a. EXISTING MODEL - My review of the Existing Model consisted of a general overview. Since the design is based on the Post Model (discussed below) there was no reason to do a complete review of the Existing Model.

       There are several split flow cards (flow leaving the model) an I need a little more time to follow the continuity of all of it. If flow leaves the system, the downstream flows should be lower than the HEC-1 runs. One result of the split flow runs is that the HEC-2 flow rates do not match the HEC-1 model.

       I did notice that the station identified in the TN card (a comment card for the NS and NG cards) was 9316 and the station in the corresponding NS card was 9129. That pattern in the other cards is that these stations match. **Schaaf & Wheeler should check as to which station it should be.**

       Section number 9286 is called out in the first TN record for the Matadero HEC-2 run. This section does not exist in the model.

    b. POST MODEL - This model does not include split flow cards. The assumption must be that no flow escapes the drainage system as did in the Existing model.

    c. In checking the flows against the HEC-1 model, I found that the flow rates were rounded up and down in the HEC-2 Model. Most of the rounding is insignificant.

    d. I reviewed the as-built Matadero Creek construction plans and have the following comments:

       1. The stations in the model do not always match the stations on the plans making exact comparison difficult. I created a table for myself using the channel lengths and station 101+62.82 for Section 9336 (downstream face of the Frontage Road). That table is included in an Excel spread sheet which I can forward to you if you want. In comparing this table to the plans, the calculated stations are pretty close. In the spreadsheet I have adjusted the

stationing occasionally, especially at each bridge crossing, to better match the plan stations and review the cross section placement for the bridge modeling.

2. Foot Bridge, Frontage Road, Hwy 101, and Bayshore Road

   a) The bridges at the downstream end of the model were each modeled individually. In looking at the low chord elevations for the bridges and comparing them to the modeled WSE's, I found that the WSE does not touch the bridge soffits. The creek under the bridges has been completely lined and the bridges do not have any piers. Under these conditions the creek in this area could be modeled as an open channel and the bridge modeling could be left out.

   In the HEC-2 and HEC-RAS manuals, the topic of parallel bridges is discussed in the bridge modeling chapter. In the discussion, it is stated that when the bridges are close together (as is the case for the first three bridges in the model) the bridges should be modeled as one bridge if they are similar in cross section (i.e. abutments and piers line up as is typical). If they are not modeled as one bridge, the bridge losses may be exaggerated because the program calculates expansion and contraction losses at every bridge. If they are "lined up" and close, there may actually not be that much expansion and contraction. Therefore, the modeling of the bridges could over estimate the actual energy losses in this reach.

   The modeling could be rerun with this section modeled as an open channel or as one bridge. **My preference would be to model as an open channel and take an uncertainty of the bridge modeling accuracy out of the model. This would also allow including some of the nuances of the channel cross section under the bridges such as where the pipe outlets effect the bench under the bridge.**

3. I reviewed the cross sections for the bridges mentioned in Number 2 above.

   a) **It appears that the low flow or fish channel was not included in the cross section. It also appeared that some of the cross sections did not exactly match the as-builts for the Matadero Creek Phase I.**

   b) The contraction and expansion coefficients used are those used for gradual transitions and are appropriate.

   For your information the Coefficients in the HEC documents are as follows

Condition	C_c	C_e
No transition loss computed	0.0	0.0
Gradual Transition	0.1	0.3
Typical Bridge sections	0.3	0.5
Abrupt Transitions	0.6	0.8

4. Greer Road Bridge

   a) The model uses a multiplication factor of 0.983 for the both the up- and downstream cross section stations. What this multiplier does is expand or contract the cross section stations by that multiplier keeping the elevations the same and the first station the same.

One use of the multiplier is to quickly widen or narrow the cross section to make fine tuned adjustments while determining the most economical cross section.

I compared the cross sections in the GR records with the as-builts and they seem to be fine without the multiplier. This bridge should be checked.

b) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

c) The bridge openings are submerged and HEC-2 documentation states that the bridge losses are computed using an orifice equation where the exit (expansion) loss coefficient is set equal to 1.0 under pressure flow conditions.

5. Louis Road Bridge

a) As with the Greer Road Bridge, the model uses a multiplication factor. In this case the factor is 0.989 for the both the up- and downstream cross section stations.

I compared the cross sections in the GR records with the proposed cross section for Louise Road in the Draft Engineer's Report and they seem to be narrower than the engineer's report. The multiplier further reduces the width of the cross section to be 1.22 feet narrower than the Engineer's Report cross section. This bridge should be checked.

b) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

c) This bridge is also under pressure.

6. Ross Road Bridge

a) The cross section downstream of the bridge is relatively close compared to the way the other bridges were modeled. They appear to be placed at the limits of work. To model the expansion losses the usual practice is to set the downstream cross section 4X the bridge opening downstream from the cross section that is right at the bridge face. This has been the practice for years and is a generally accepted practice though there are new approaches in debate that tend to reduce the distance from the bridge to the up- and downstream cross sections. Any inaccuracy caused by setting the cross section too close to the bridge would underestimate the bridge expansion losses. Some study could be done at this bridge. I do not know if the water surface elevations would change that much if the model were refined. Any inaccuracy this may cause in the contraction loss would be minimal given that the cross sections are virtually the same.

b) As with the Greer Road and Louise Road Bridges, the model uses a multiplication factor. In this case the factor is 0.968 for the both the up- and downstream cross section stations.

I compared the cross sections in the GR records with the as-built cross section for the bridge and found that the overall top of bank to top of bank width is 2.0 feet wider in the model that on the plans WITH the multiplier and the bottom width is 0.62 feet width in the model than on the plans WITH the multiplier. This bridge should be checked.

c) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

d) This bridge is also under pressure.

7. Middlefield Road Bridge

a) There are two cross sections at the upstream side that are very close together. Any inaccuracy this may cause in the contraction loss would be minimal given that the cross sections are virtually the same. (See comments on Ross Road Bridge)

b) As with the Greer Road and Louise Road Bridges, the model uses a multiplication factor. In this case the factor is 0.991 for the both the up- and downstream cross section stations.

I compared the cross sections in the GR records with the as-built cross section for the bridge and found that the overall top of bank to top of bank width is 1.17 and 1.56 feet wider in the model that on the plans for the downstream and upstream cross sections respectively. The bottom widths did not match the as-builts either. This bridge should be checked.

c) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

d) This bridge is also under pressure.

8. Cowper Street Bridge

a) The two cross sections at the upstream side that are very close together. Any inaccuracy this may cause in the contraction loss would be minimal given that the cross sections are virtually the same. (See comments on Ross Road Bridge).

b) At this bridge the model copies the cross section of the downstream face and uses it for the upstream face. The slope of the channel is relatively small (0.00215 ft/ft) so this may be ok. A more diligent modeler would have at adjusted the flow line to match the as-builts.

c) I compared the cross section to the as-builts and the bottom width is 10.7 in the model and 13.0 feet wide on the as-builts. This bridge should be checked.

d) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

e) This bridge is also under pressure.

9. Waverely Street Bridge

a) The two cross sections at the upstream side that are very close together. Any inaccuracy this may cause in the contraction loss would be minimal given that the cross sections are virtually the same. (See comments on Ross Road Bridge).

b) At this bridge the model copies the cross section of the downstream face and uses it for the upstream face. The slope of the channel is relatively small (0.00215 ft/ft) so this may be ok. A more diligent modeler would have at adjusted the flow line to match the as-builts.

c) I compared the cross section to the as-builts and the bottom width is 12.8 in the model and 13.0 feet wide on the as-builts. **This bridge should be checked.**

d) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

e) This bridge is also under pressure.

10. Alma/SPRR Culvert

a) **The two cross sections at the downstream side that are very close together. The change in cross section at the downstream side should be looked at. The expansion and contraction coefficients do not seem to be high enough.**

b) **As Chuck Anderson pointed out, there is a48" RCP at the downstream discharging downstream. The channel could be modified to reduece losses here by using a more gradual transition. Conforming the 48" outlet would be a challenge, but would be feasible.**

c) I compared the cross section to the as-builts and they look ok.

d) The contraction and expansion coefficients are appropriate for the warp transition and are lower through the bridge.

e) This culvert is inlet controlled.

11. Reach Lengths, Warnings & Cautions

Most of the reach lengths throughout the model appear the be reasonably spaced. There are a number of Warnings regarding the K-Ratio throughout the model. These warnings are usually eliminated by adding cross sections so that the reach lengths are not as long. There are times when the K-Ratio warnings cannot be eliminated because the flow regime is rapidly varied. The warnings are distinguished from errors in that they are to indicate to the modeler that there may be a need to refine the model to improve its accuracy.

Cautions are similar to warnings, but are to be taken with some greater concern. For example, the cautions for consecutive sections 29917, 30042, and 30055 relate to the program assuming critical depth. This could mean that the flow in this area is supercritical and that the modeling for this reach should be reviewed. More cross sections may remove these cautions or modeling this portion assuming super critical flow may result in removing them and finding that the flow is in fact supercritical. This would be especially important if the depth is very important or erosion velocities are a concern.

From these warnings it appears that he flow just downstream of Ross and Middlefield are very near critical depth. The model assumed critical depth in several areas, but not continuously. Running the HEC-2 model to check for critical depth may not be worthwhile. If the creek were remodeled using HEC-RAS, this check would be simple. The main issues that would be revealed if the model were check for critical depth would be that more accurate velocities would be found. Supercritical flow would have higher velocities. Potential scour of the lined channels could be addressed with more accurate velocities.

If we were to convert the Model to HEC-RAS, we should check for supercritical consitions.

e. General HEC-2 review comments

1. **Several of the bridge models do not match the as-builts or Draft Engineer's Report. At a minimum, the HEC-2 model should be revised to adjust those bridges to match the geometry of the as-builts. Additional changes such as eliminating the bridge modeling for the bridges at and around HYW 101 would be worthwhile if we were to revise the model.**
2. Converting the Model to HEC-RAS would take additional time and effort. This is mainly because the bridge portions do not directly import into HEC-RAS that well and each bridge would have to be remodeled in HEC-RAS. Since only one bridge has splitter walls, and because we do not expect flow over the bridges, the bridge modeling would be pretty straight forward.

   HEC-RAS also uses different methods of modeling. Some jurisdictions do not like the new methods. FEMA did not, but now does accept HEC-RAS models for Flood Insurance Studies. I think, though, that we would have to show that the HEC-RAS Model for the existing conditions match the HEC-2 model before they would except the HEC-RAS model as the new FIS model. We could check into this requirement if the Water District wanted us to pursue it.

13. Review of FORM 5 - There is not much to review in FORM 5.
14. Review of FORM 6 - It appears that Schaaf & Wheeler's task for some of FORM 6 was to pull together information from reports by others and package it for the CLOMR. There are also some straight forward sediment transport calculations that appear to be in order.
15. Review of FORM 7 - FORM 7 contains several forms related to modification to proposed modification to bridges including Greer Road, Louis Road, Ross Road, Middlefield Road, Cowper Road, Waverley Street, Alma Street/Southern Pacific RR, Park Boulevard, Lambert Avenue, and El Camino Real. The box indicating that the revision (CLOMR submittal) reflects a modified bridge previously modeled in the Flood Insurance Study (FIS). Chuck Anderson of Schaff & Wheeler informed us that the modifications were done in the model to account for changes done since the FIS was completed.

If you have any questions regarding these comments, please contact me.

APPENDIX 2

GEOTECHNICAL DESIGN REVIEW

PARIKH CONSULTANTS, INC.
481 Valley Way
Milpitas, Ca 95035
Phone: (408) 945-1011

Matadero Creek Remediation Project
Comment on report titled "Geotechnical Investigation Matadero Creek Remediation Project (Report No. 1453-1)" dated September 27, 1999 prepared by Lowney Associates

The following comments were based on the above report:

(1) Exploratory borings by others (1983-1990)

(a) The "Site Plan" shows the approximate location of these borings. Are these boring logs available? If so, they should also be attached in the report for information.

(2) Page 23 – Drilled Pier Foundation for Floodwalls

(a) Reference or method to derive the allowable skin friction in compression and uplift should be described.

(b) If spread footing is recommended (according to Mohammad Khan's comment dated November 13, 2001), seepage analysis should be carried out to investigate the uplift pressure on the floodwall due to seepage and exit gradient for any possibility of "quick" condition. If seepage is anticipated to be a problem, seepage control of the floodwall should be recommended in the report.

(3) Page 25 and 26 – U-Frame Section Channel

(a) Other than the foundations and lateral earth pressure, the "uplift resistance" of the U-Frame Section Channel should also be addressed. The U-Frame Channel will be positioned up to 3 feet below the existing channel. Groundwater was measured at 7.5 to 13.5 feet below the existing ground surface, uplift pressure on the U-Frame Channel is anticipated to be significant.

(4) Page 26 – Lateral Earth Pressures

(a) The report stated "we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H psf, where H is the distance in feet between the top of the footing and the top of the wall." Does it mean trapezoidal load distribution for the at-rest soil condition? How was 8H obtained? Please clarify.

...../2

Matadero Creek Remediation Project
Comment on report titled "Geotechnical Investigation Matadero Creek Remediation Project (Report No. 1453-1)" dated September 27, 1999 prepared by Lowney Associates (Continued)

(5) Page 29 – Foundation Options for Louis Road Bridge

(a) The report recommended "the CIDH piles extend at least 30 feet below the bottom of the pile cap for 45 ton piles, and 40 feet below the bottom of the pile cap for 70 ton piles. These recommendations are based on the assumption that the pile cap bottom is located no deeper than 5 feet from the existing or proposed ground surface", our comments are:

(i) Justification with calculations is required for the vertical pile capacity.

(ii) There is an increase in 25 tons in vertical pile capacity with an increase in 10 feet of pile length, is it due to abrupt increase in soil strength with depth. Soil profile and strength parameters should be developed in the calculations of the pile capacity. Cut-off and tip elevations of the pile should be specified in the report.

(iii) What is the significance of "no deeper than 5 feet from the existing or proposed ground surface" in the pile design?

(iv) The nearest borings to Louis Road are LA-3 and LA-4. The depth of these borings is 30 feet. For the 70 ton pile with the approximate depth of 45 feet, a deeper boring may be necessary for confirmation of ground profile in the calculation of pile capacity (both vertical and lateral).

(b) The corrosive potential due to the native soil on the foundation piles should also be investigated.

(6) Page 30 – Foundation Options for Louis Road Bridge

(a) The computer program used for the lateral load analyses should be described in the report.

(b) Soil profile and strength parameters (such as soil modulus parameters and soil strain parameters) should be developed in the estimation of the pile response under lateral load.

Matadero Creek Remediation Project
Comment on Final Report titled "Geotechnical Investigation Matadero Creek Remediation Project (Report No. 1453-1)" dated December 21, 2001 prepared by Lowney Associates

The following comments were based on the above report:

(1) Page 7 – Ground Water

(a) The ground water depth and corresponding NGVD 29 elevations should be further checked. Some of the numbers appear to be not right.

(2) Page 30 – Spread Footing Foundations for Floodwalls

(a) Whether the friction between the footings and the supporting grade can be combined with the passive resistance in the calculation of lateral earth resistance should be stated.

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MEMORANDUM

July 1, 2002
Job No: 201147.10

To: Mr. Steve Machida
Mark Thomas & Co.
90 Archer Street
San Jose, Ca 95112

From: Gary Parikh, P.E., G. E. 666
Parikh Consultants, Inc.

Sub: Matadero Creek Overflow Bypass Channel Project
Review Comments on Draft Soil Report dated September 27, 2001 (Report No. 1453-G)

The following are our comments on the subject report:

(a) The "Dewatering" recommendations state that all dewatering systems be properly designed to prevent pumping soil fines with the discharged water. The contractor should sample and test the groundwater for soil fines content from the wells, as needed.

(b) The soil-structure interaction of the interlocking block walls should be more elaborated in the report. Who will do the design? Are the blocks just a facing? Are the blocks providing lateral capacity and if so, what would happen if they shift?

(c) Are the proposed interlocking sheet piles temporary? If the contractor wants them to be removed the concern is for the voids left behind by their removal.

(d) The interlocking block wall has a permeable material for drainage. In our opinion even with this material it is prudent to use a filter fabric to protect the fines from plugging the backfill and/or washing away of the fines due to "suction effect" from the water flow.

(e) Backfill compaction will require some heavy equipment. It may be prudent to use a backfill material that is "self compacting" or requires minimal efforts. This could be similar to crushed rock.

(f) The project description discusses use of an interlocking block type gravity wall. However, the "backfill" section discusses placement of wall reinforcement and its length. This generally relates to a MSE (Mechanically Stabilized Embankment) type of wall, which is internally stable. This should be clarified.

(g) Any proposed over-excavation at the bottom of the bypass channel should be considered in the slope stability evaluation.

Please call if you have any questions on the above.

356 S. Milpitas Blvd., Milpitas, CA 95035-5421 • (408) 945-1011 • Fax: (408) 945-1012 • www.PARIKHNET.com
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APPENDIX 3

HAZARDOUS MATERIAL REVIEW

GEOCON
CONSULTANTS, INC.
ENVIRONMENTAL ■ GEOTECHNICAL ■ MATERIALS

GEOCON
CONSULTANTS, INC.
ENVIRONMENTAL ■ GEOTECHNICAL ■ MATERIALS

Project No. E8101-06-01
November 9, 2001

Mr. Gary Parikh
Parikh Consultants, Inc.
481 Valley Way, Building 1
Milpitas, California 95035

Subject: DOCUMENT REVIEW
MATADERO CREEK REMEDIATION PROJECT, PALO ALTO, CALIFORNIA

Dear Mr. Parikh:

As you requested, Geocon Consultants, Inc. (Geocon) has reviewed the two documents you provided for the Matadero Creek Remediation Project in Palo Alto, California. The documents reviewed are:

- *Phase I Environmental Site Assessment and Soil and Ground Water Quality Screening, Matadero Creek Remediation Project, Palo Alto, California* (hereinafter referred to as "Phase I" report). Prepared by Lowney Associates, July 15, 1999.
- *Additional Soil and Ground Water Quality Evaluation, Matadero Creek Remediation Project, Palo Alto, California* (hereinafter referred to as "Additional Evaluation" report). Prepared by Lowney Associates, August 1, 2001.

Note that additional project documents are referenced in the Additional Evaluation report but were not provided for review. The project documents not provided for Geocon's review include:

- *Limited Geotechnical and Hazardous Materials Evaluation*. Unpublished report prepared by Lowney Associates, November 24, 1999.
- *Soil and Ground Water Quality Evaluation, Matadero Creek Remediation Project*. Unpublished report prepared by Lowney Associates, March 15, 2000.

Phase I Report

1) The Phase I report indicates that an ARCO service station is located at 2995 Middlefield Road (adjacent to the south bank of Matadero Creek). In August 1988, soils impacted with petroleum hydrocarbons were encountered during excavation activities associated with a creek improvement project. The source of the petroleum hydrocarbon impacts was presumed to be the former leaking underground storage tank (UST) at the ARCO service station. The Phase I report indicates that residual levels of petroleum hydrocarbons may be encountered during creek channel expansion. In addition to residual petroleum hydrocarbons in soil, Geocon is concerned that petroleum hydrocarbon impacted groundwater may possibly be encountered during construction. Geocon recommends that the construction contractor be advised of this possibility and that appropriate contingencies be established at the beginning of the project in order to reduce potential contractor change orders.

2) Table 1 of the Phase I report identifies two sites (842 Wintergreen Avenue and Megher Property) that were listed in the EDR Database Report (Phase I report - Appendix C). It is not clear why these properties were not further discussed or investigated.

3) The EDR Database Report (Phase I report – Appendix C) identifies two additional sites (Piers Dairy and Old Piers Dairy) that appear to be adjacent to Matadero Creek. It is not clear how these properties are/were used and whether the potential for these dairy properties to adversely affect water quality (i.e., elevated nitrates from dairy liquid and solid waste) has been addressed or is of concern to the project.

Additional Evaluation Report

1) In Section 1.1 of the Additional Evaluation report, the acronym "PAMSC" is defined as the Matadero Creek Remediation Project. Geocon assumes that the correct definition of PAMSC is Palo Alto Maintenance Service Center, which is located adjacent to Matadero Creek.

2) Based on the groundwater sampling results, it was concluded that petroleum hydrocarbon impacted groundwater may be encountered adjacent to PAMSC during creek construction activities. The Additional Evaluation report recommends that groundwater extracted from within 75 feet of the PAMSC tank complex should be assumed to be impacted and appropriately disposed. While Geocon agrees that impacted groundwater may be encountered, Geocon recommends that no distance from the PAMSC tank complex be used to limit the extent of potentially impacted groundwater. Establishing a fixed distance may allow for increased contractor change orders in the event that the lateral extent of impacted groundwater exceeds 75 feet from the tank complex.

Please contact me should you have any questions regarding our review of the provided project documents.

Sincerely,

GEOCON CONSULTANTS, INC.

Richard W. Day, CEG, CHG
Regional Manager

RWD:rwd

APPENDIX 4

STRUCTURAL REVIEW

MARK THOMAS & COMPANY, INC.
Providing Engineering, Surveying and Planning Services

MATADERO/BARRON CREEKS LONG-TERM REMEDIATION PROJECT

REVIEW OF PRELIMINARY STRUCTURAL INVESTIGATION

1. INTRODUCTION

This document outlines Mark Thomas & Co., Inc.'s (MTCo) review of the "Structural Investigation Report - June 2001" for the Matadero/Barron Creeks Remediation Project as prepared by Biggs Cardosa Associates for the Santa Clara Valley Water District.

This outline of MTCo's review focuses primarily on the findings that were not congruent with the structural investigation report. The nomenclature used here correlates directly with that used in the structural investigation report. A summary of the existing floodwall types is included in the appendix of this report. Also included is a location map which shows each Reach segment and bridge location.

2. SCOPE

The review of the structural investigation report was limited to the structural analysis of the existing structures and proposed improvements as presented in the report and supporting calculations.

3. SUMMARY OF REVIEW

A. Design Criteria

The preliminary analysis design criteria used in the structural investigation report for the floodwalls at Reach 2 (Bayshore Freeway to Greer Rd) did not appear to consider FEMA seepage requirements. In discussions with our Geotechnical Engineer, Parikh Consultants, it was agreed that seepage might be an issue in the analysis and design of the floodwalls along the Reach 2 portion of this project where the wall footings are not protected by channel lining.

If seepage is determined to be an issue during the design process, the proposed wall replacement details shown in the report will need to be altered to incorporate a seepage control measure such as a cutoff wall. If this is the case, a complete replacement floodwall may be required, adding a significant cost to the floodwalls along Reach 2.

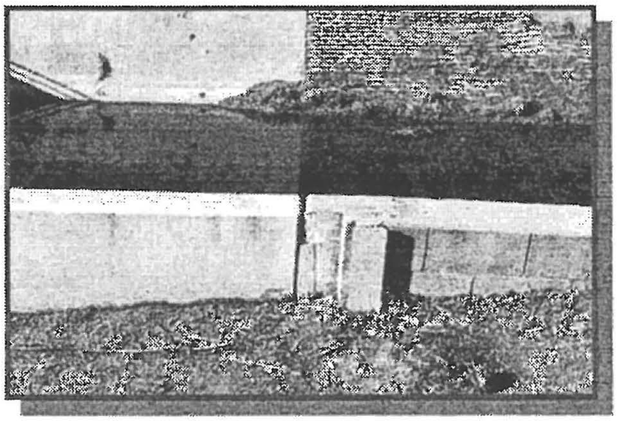
The design criteria used for the bridge investigation are suitable for this project.

B. Overflow Bypass

Reach 1 includes plans for the construction of a concrete overflow bypass from the Palo Alto Flood Basin to the Bayshore Freeway. The preliminary design concepts proposed in the report are reasonable and workable solutions. MTCo will perform a detailed review when the final alternative for the overflow bypass is determined.

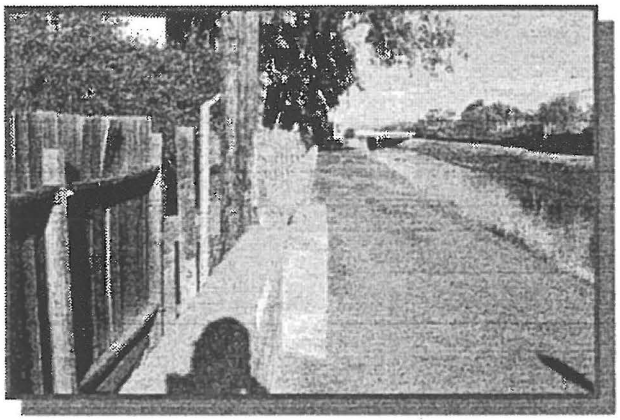
C. Floodwall Replacement - Reach 2 (Bayshore Fwy to Greer Rd)

The Reach 2 segment of this project is primarily a sack concrete slope-lined channel with a continuous earthen maintenance road on each bank. In the structural investigation report it was stated that the Type I floodwall had not been constructed, but had been instead been substituted with a Type III floodwall. During review of the as-built plans and field verification performed by MTCo it was discovered that the Type I walls had been built as shown in the as-built plans and were not substituted with the Type III wall as described in the structural investigation report. However, preliminary analysis performed by MTCo has determined that these walls are still likely to remain under the "Buttress or replace floodwall" improvement category as stated in the report.



'Type I' wall type at Matadero Creek Reach 2

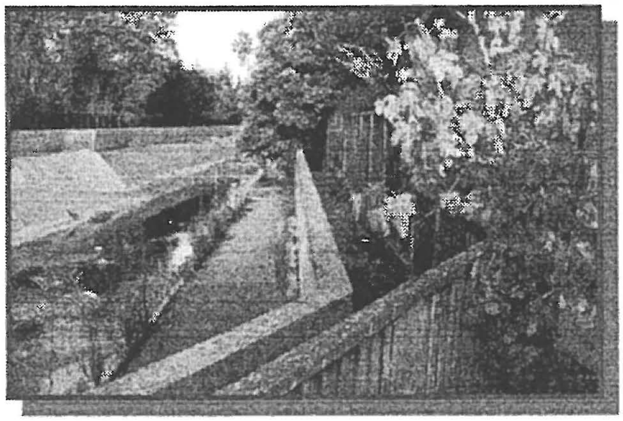
The structural investigation report describes the Existing and Modified wall types as being on spread footings. MTCo's review of the as-built plans and field verification determined that the Existing/Modified wall types were built with steel posts at 8' on center using a detail similar to the Type I walls.



'Type I' wall type at Matadero Creek Reach 2

Preliminary analysis performed by MTCo has determined that these walls will most likely continue to fall under the "Buttress or replace floodwall" improvement category.

The structural investigation report also stated that the Type IV floodwall had not been used at all on the previous projects. During review of the as-built plans and field verification performed by MTCo it was determined that the Type IV wall had not only been built along Reach 3 adjacent to Louis Rd. It was also determined that the length of this portion of wall is significantly longer than shown in the as-built plans. Preliminary analysis performed by MTCo has determined that these walls will also fall under the "Buttress or replace floodwall" improvement category.



'Type IV' Floodwall at Matadero Creek Reach 3

D. Floodwall Improvements - Reach 3 through Reach 9 (Greer Rd to Alma St)

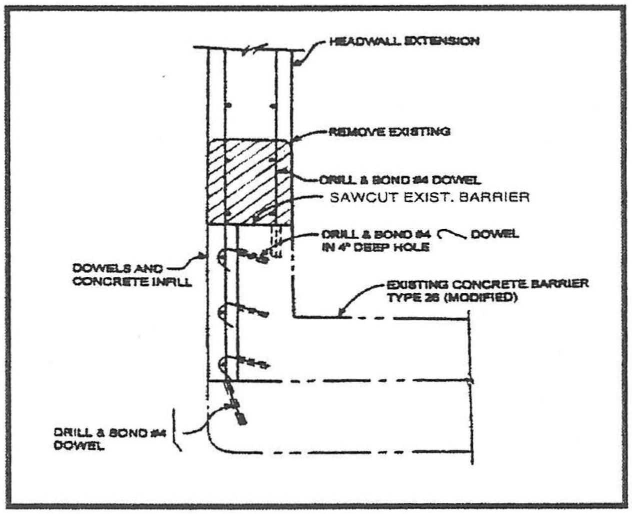
At the lined channel sections of Reaches 3 through 9, the effects of seepage do not need to be considered because the channel lining protects against the possibility of seepage. Floodwall extensions will be able to be designed as suggested using either the replacement or buttress details as presented in the report.

E. Headwall Improvements

In reviewing the proposed headwall improvements it was determined that the headwall to soffit dimension for Greer Rd shown in the structural investigation report is correct, but the required extension to achieve this is actually 2' greater than the 3' extension described in the report. This will result in an upstream headwall height of approximately 8' above the deck level on the Greer Road Bridge.

MTCo believes it will be possible to achieve the 8' headwall height at Greer Bridge, but this will require significant modification or possible replacement of the existing headwall.

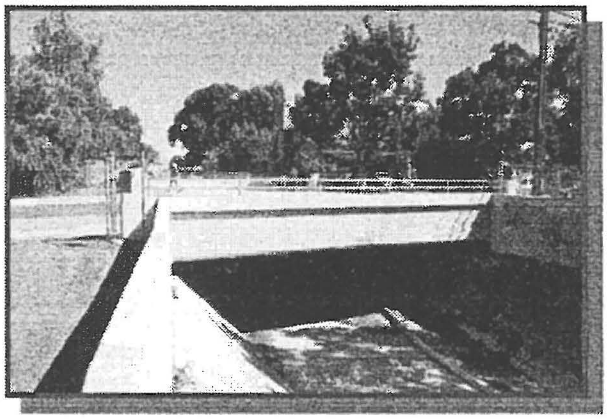
MTCo believes it will be possible to extend the existing headwalls on all the other bridges, but would suggest a more integrated version of the detail shown in the structural investigation report should be used.


HEADWALL EXTENSION
REMOVE EXISTING
DRILL & BOND #4 DOWEL
SAWCUT EXIST. BARRIER
DRILL & BOND #4 DOWEL IN 4" DEEP HOLE
DOWELS AND CONCRETE INFILL
EXISTING CONCRETE BARRIER TYPE 26 (MODIFIED)
DRILL & BOND #4 DOWEL

Modified Headwall Extension Detail

F. Bridge Structures

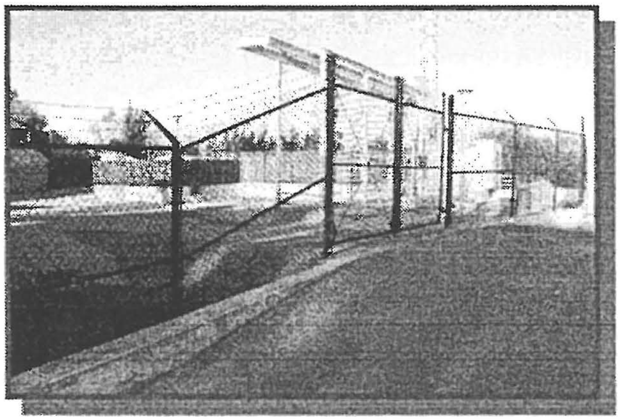
The D/C ratios in the structural investigation report for the bridge structures along Matadero Creek lead to the conclusions made in the report. Although MTCo could not verify the D/C ratios in reviewing the calculations that accompanied the report, preliminary analysis performed by MTCo resulted in D/C ratios that support the conclusions drawn in the report. MTCo concurs with the decision to replace Louis Road Bridge.



Louis Road Bridge

G. Maintenance Access Ramps

The structural investigation report offers two solutions for providing flood protection at the maintenance access ramps: raising the ramp elevation (resulting in an increase in the ramp slope) or seasonal floodgates. It is agreed that if the ramp elevations are raised, the slope of the ramps may become a problem. This is especially true at the Reach 2 pump station where the slope of the existing AC ramp is already at 10%.



Maintenance Access Ramp at Matadero Creek Reach 2 Pump Station

It is apparent from the Draft Engineers Report issued by the Santa Clara Valley Water District that a system of permanent, removable aluminum gates is proposed for flood control at the maintenance ramps. The system described seems to be a viable solution and would eliminate the need for increasing the ramp slopes.

4. CONCLUSIONS

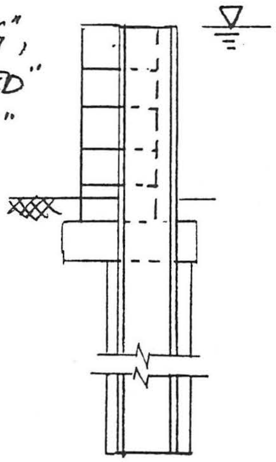
- Preliminary design concepts for the concrete overflow bypass at Reach I are workable solutions.
- Seepage may be an issue for the Reach 2 floodwalls. May need to incorporate seepage control measures in floodwall details.
- MTCo's field review revealed several discrepancies between the field conditions and the assumptions made in the structural investigation report, however, the conclusions reached for these wall segments remain unchanged.
- Headwall height at Greer Bridge will be approximately 2' greater than described in the report. An alternative headwall modification detail or replacement of the existing headwall is recommended.
- Conclusions about the bridge structures presented in the report seem appropriate.
- A system of permanent, removable aluminum gates is a viable solution for modifying the maintenance access ramps.

APPENDIX

MARK THOMAS & CO. INC.
ENGINEERS ⊕ SURVEYORS ⊕ PLANNERS

FILE NO.
JOB TITLE MATADERO CREEK
SUBJECT FLOODWALL INDEX
PREPARED BY TW DATE
CHECKED BY DATE

"EXISTING",
"MODIFIED"
& "TYPE I"



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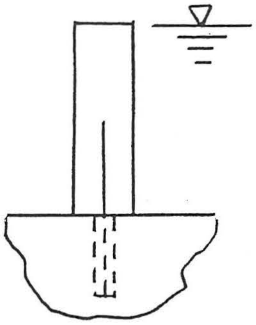

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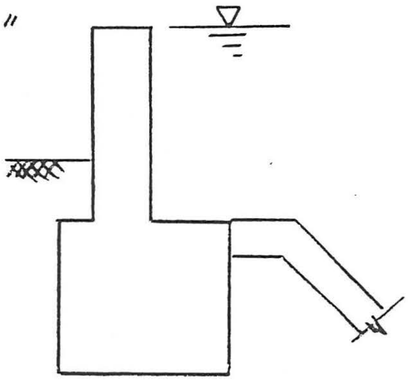

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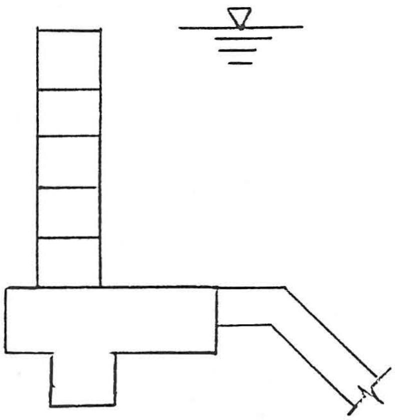

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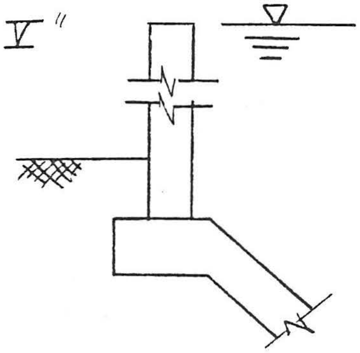

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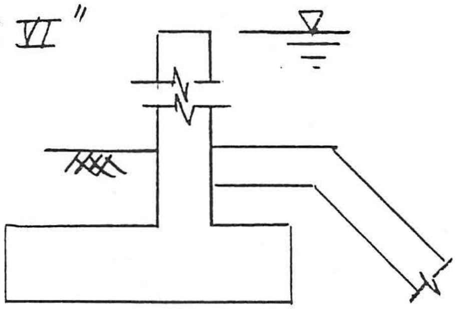

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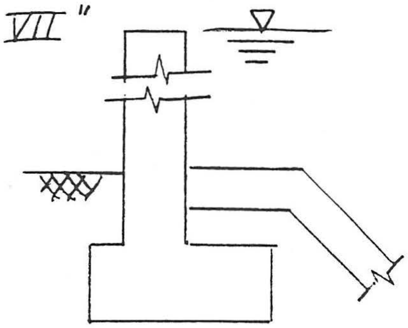

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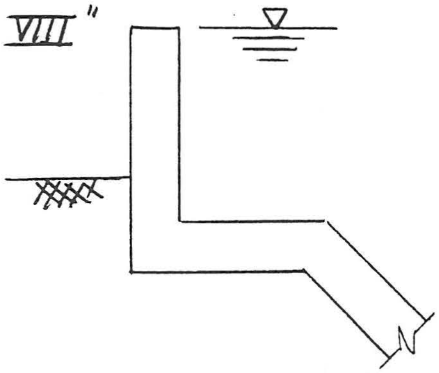
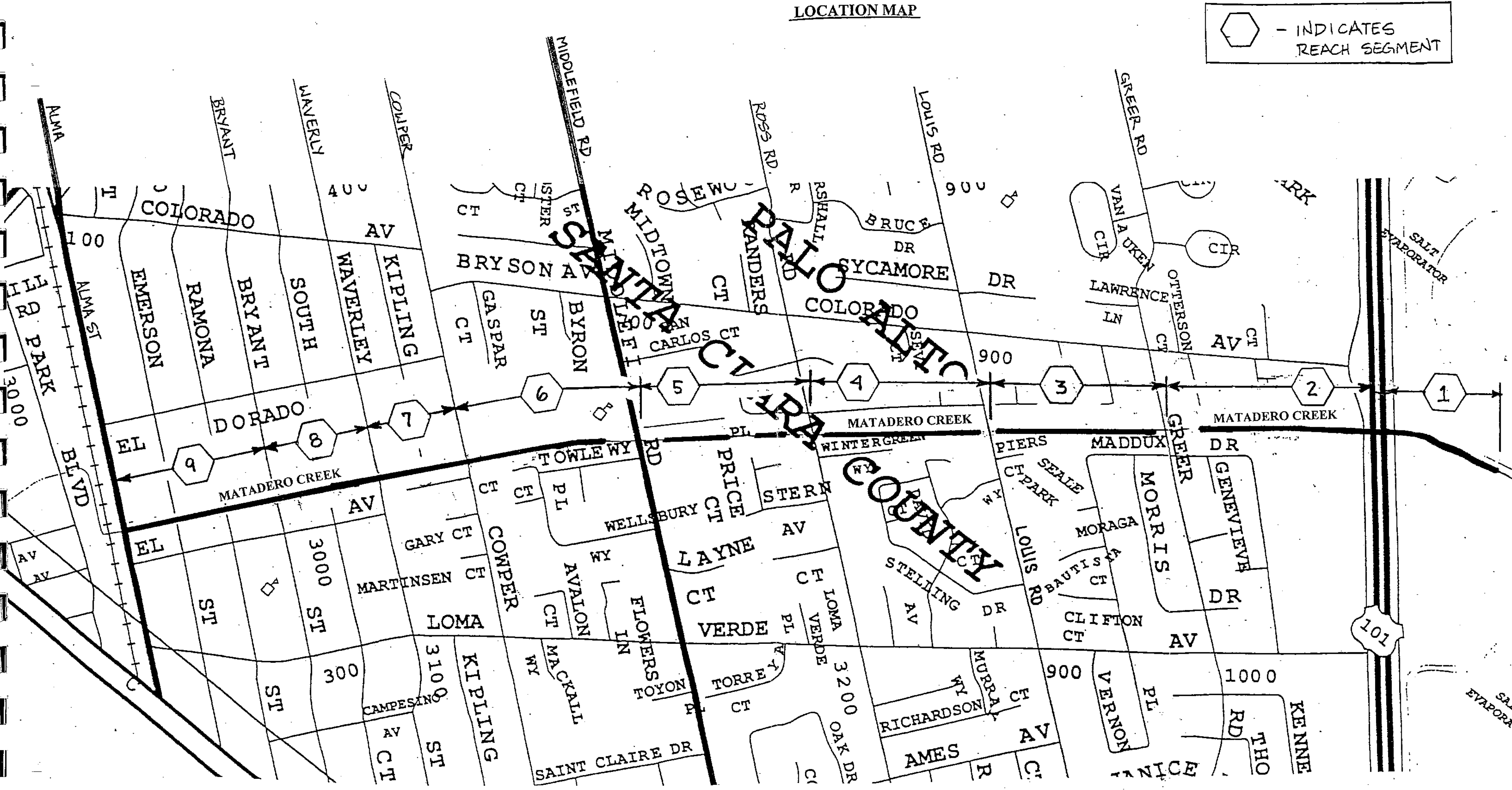


MATADERO/BARRON CREEKS LONG-TERM REMEDIATION PROJECT

LOCATION MAP



APPENDIX 5

EIR/ER REVIEW

MARK THOMAS & COMPANY, INC.
Providing Engineering, Surveying and Planning Services

MARK THOMAS & COMPANY, INC.
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PRINCIPALS
RICHARD K. TANAKA
PHILLIP R. SAVIO
TIMOTHY R. FLEMING
DAVID E. ROSS
MICHAEL J. LOHMAN

November 30, 2001

File No. 22-51004
Letter #25

Mr. Eric Tsou
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, California 95118

Re: Matadero Creek Remediation Project
Draft EIR/ER Comments

Dear Mr. Tsou:

Mark Thomas & Co. has reviewed the draft Environmental Impact Report/Engineer's Report and in general, there are a few inconsistency issues with respect to the report. The front portion of the EIR recommends how to achieve the project objectives. However, some of the items in this portion are either not clear or do not coincide with the conclusion of the engineering study in Appendix B. The specific comments are as follows:

- One of the main objectives of the project is to "obtain a letter of map revision from FEMA to remove the existing flood prone areas from the Flood Insurance Rate Map…". However, the Engineer's report (Page 4-8) mentions that since the flood basin levees do not meet FEMA standard and will not be upgraded to conform to this standard through this particular project, "areas currently mapped within the one-percent tidal floodplain will remain so mapped". There is some confusion between what areas are being helped by this project.

- Page 6 of the EIR describes the channel modifications upstream and downstream of Louis Road. The modifications include replacing the trapezoidal channel with a U-shaped channel extending approximately 65 feet under Louis Bridge, a 40-foot transition upstream and a 20-foot transition downstream (125 feet total). Later in the report, on page 61, the modification is specified as approximately 120 feet in length.

- The EIR states that the bypass channel would be constructed within the District's right of way along the North and Southeast edges of the MSC (page 5). However, on some of the figures, it appears as if the bypass channel may cut into the MSC property on the NE corner (Figure 10 on page 74). The engineer's report also states that this bypass channel will require 0.16-acre right-of-way acquisition from the MSC.

- Under Enhancement No. 1 on page 12, the EIR states that the "proposed bypass channel requires a 3-foot tall floodwall on the western edge of the channel". However, this requirement is not mentioned elsewhere (such as on page 5 or 6-14) when describing this alternative. On pages 6-8 and 7-5 of the Engineer's report a proposed vertical wall or levee around the MSC is described, but the connection between the two statements is unclear.

- Would the enhancement trail, benches and vegetation/visual enhancements be placed within the 10-foot construction buffer zone of the proposed bypass channel? The EIR states that it will be placed "between the floodwall and the fence of the MSC boundary" (page 12). However, Figures 4 on page 8 and 6-5 on page 6-15 shows the floodwall abutting the MSC and does not show the construction buffer zone.

- The overflow bypass channel cross-sections shown in the EIR are inconsistent. In general, the bypass channel is described to be 60 feet in width, with a 10-foot construction buffer on each side. However, Figure 6-5 on page 6-15 shows a cross-section with a width of 58 feet and does not show the construction buffer zone. Figure 4 on page 8 of the EIR, while nearly identical to Figure 6-5, shows a bypass channel width of 60 feet. In addition, Figure 7-2 on page 7-6 shows a 50-foot channel section with a 10-12 foot construction zone, but does not show a floodwall.

- There is some confusion between two separate statements in the EIR regarding relaxing FEMA's freeboard requirements. Page 9 of the EIR states that "If the freeboard criteria can be relaxed, the required wall heights between Greer Road and Louis Road might be reduced to 5.3 feet". The statement on page 99 is "If the freeboard criteria can be relaxed, the required wall heights could be reduced by up to 5.3 feet in the most optimistic case." To 5.3 feet and up to 5.3 feet are very different statements.

- The EIR doesn't mention anything about the new warped transition that will be constructed on the south channel back at the downstream face of the Alma Street/SPRR culvert and the improvements to an existing 84-inch diameter storm drain outfall as part of the project. The first time it is mentioned is in the Engineer's report (page 7-1).

If you have any questions or require further information, please call me.

MARK THOMAS AND CO. INC

Stan Mm

Steven Machida

cc: Richard Tanaka